भारतीय मानक
Indian Standard

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(Reaffirmed 2020)

उच्च ऑगी अधिप्रवाह और ऑरिफिस उत्प्लावों की हाइड्रोलिक डिज़ाइन के लिए सिफारिशें

(*दूसरा पुनरीक्षण*)

Hydraulic Design of High Ogee Overflow and Orifice Spillways — Recommendations

(Second Revision)

ICS 93.16

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Dams and Spillways Sectional Committee, WRD 09

FOREWORD

This Indian Standard (Second Revision) was adopted by the Bureau of Indian Standards, after the draft finalized by the Dams and Spillways Sectional Committee had been approved by the Water Resources Division Council.

Spillways are devices provided in conjunction with dams to pass surplus water for reservoir regulation and safety. Various types of spillways include overflow, shaft or morning glory, siphon, chute, side channel, tunnel spillway, etc. The overflow type is by far the most common one. The usual form of overflow spillway has a rounded crest with an ogee profile.

This standard was first published in 1973 and revised in 1998. This revision incorporates the latest practices being followed in the field, the major changes being in clause **6** dealing with ogee profile for spillway with breast wall.

For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2 : I960 'Rules for rounding off numerical values (*revised*)'. The number of significant places retained in the rounded off values should be the same as that of the specified value in this standard.

Indian Standard

HYDRAULIC DESIGN OF HIGH OGEE OVERFLOW AND ORIFICE SPILLWAYS — RECOMMENDATIONS

(Second Revision)

1 SCOPE

This standard recommends criteria to be adopted for hydraulic design of high ogee overflow and orifice spillways, applicable to spillways without gates, with gates and with breast walls.

2 LETTER SYMBOLS

For the purpose of this standard, the following letter notations shall have the meaning indicated against each.

A_1,A_2, etc = horizontal dimension defining upstream quadrant of the crest,

B_1,B_2, etc = vertical dimension defining upstream quadrant of the crest,

C = non-dimensional discharge coefficient,

C_a = discharge coefficient as affected by downstream apron,

C_b = discharge coefficient for spillways with breast wall,

C_d = discharge coefficient for design head = $2/3\sqrt{2g}\ C$,

C_g = discharge coefficient for flow under the gate,

C_h = discharge coefficient for head H (other than -design head),

C_s = discharge coefficient as affected by submergence of the crest,

D = net opening for the spillway with breast wall,

G_o = gate opening

g = acceleration due to gravity,

H = head of overflow,

H_a = head due to velocity of approach,

H_c = head from reservoir level up to the centerline of the opening of the gate,

H_d = design head,

K_1,K_2, etc = variable parameters,

K_a = abutment contraction coefficient,

K_p = pier contraction coefficient,

L = effective length of overflow crest,

L' = net length of overflow crest (excluding thickness of pier),

M = riser of the crest,

N = number of piers,

n_1,n_2, etc = variable parameters,

P = height of the spillway crest measured from the river bed,

Q = discharge,

q = discharge per unit length of the spillway

R = radius of abutment,

R_g = radius of crest gate,

V_a = approach velocity,

X,X_1,X_2, Y,Y_1,Y_2,etc = co-ordinates of the profile, and

β = angle formed by the tangent to the gate lip and the tangent to the crest curve at the nearest point of the crest curve.

3 TERMINOLOGY

3.1 For the purpose of this standard, the following definitions shall apply.

3.1.1 *Ogee Spillway* — Spillway which has its overflow profile conforming, as nearly as possible, to the profile of the lower nappe of a ventilated jet of water flowing over a sharp crested weir (free overflow) or through an orifice (spillway with breast wall).

3.1.2 *High Overflow Spillway* — Overflow spillways are classified as high and low depending on whether the ratio of the height of the spillway crest measured from the river bed to the design head is greater than and equal to or less than 1.33 respectively. In the case of high overflow spillways the velocity of approach head may be considered negligible.

3.1.3 *Head* — The head is the distance measured vertically from the water surface (upstream of the commencement of drawdown) to the crest elevation. It also includes head due to velocity of approach.

3.1.4 *Design Head* — The design head is that value of head for which the ogee profile is designed.

3.1.5 *Breast Wall* — A suspended wall on top of the spillway, spanning between the piers, so as to create a rectangular opening above the crest level to pass the flow of water stored behind the wall.

4 OGEE PROFILE FOR FREE OVERFLOW

4.1 Shape of the Profile

4.1.1 The ogee profile consists of two quadrants, the upstream quadrant and the downstream quadrant. Once the design head H_d of the spillway is fixed, the crest geometry may easily be evaluated. The recommended shape is based on detailed observations of the lower nappe profile of a fully ventilated thin-plate weir. Such a profile would generally result in atmospheric pressure along the entire spillway surface at design head H_d. For head lower than H_d, the pressure would be higher than atmospheric and for higher heads, sub-atmospheric pressure would result.

4.1.2 The ogee profile is divided into three groups as follows:

a) Spillways with vertical upstream face,
b) Spillways with sloping upstream face, and
c) Spillways with crest offsets and risers.

However, the same general equation for the upstream and downstream quadrants are applicable to all the three cases as described in **4.1.3** to **4.1.5**.

4.1.3 *Spillways with Vertical Upstream Face*

4.1.3.1 *Upstream quadrant*

The upstream quadrant of the crest may conform to the ellipse:

$$\frac{X_1^2}{A_1^2} + \frac{Y_1^2}{B_1^2} = 1$$

The magnitudes of A_1 and B_1, are determined with reference to the parameter P/H_d, from the graphs, given in Fig. 1.

4.1.3.2 *Downstream profile*

The downstream profile of the crest may conform to the equation:

$$X_2^{1.85} = K_2 H_d^{0.85} Y_2$$

The magnitude of K_2, is determined with reference to the parameter P/H_d from the graphs given in Fig. 1.

4.1.4 *Spillway with Sloping Upstream Face*

In the case of sloping upstream face, the desired inclination of the face is fitted tangential to the elliptical profile described in **4.1.3.1**, with the appropriate tangent point worked out from the equation. The profile of the downstream quadrant remains unchanged.

4.1.5 *Spillways with Crest Offsets and Risers*

Whenever structural requirements permit, removal of some mass from the upstream face leading to offsets and risers as shown in Fig. 2, results in economy. The ratio of riser M to the design head H_d that is M/H_d, should be at least 0.6 or larger, for the flow conditions to be stable. The crest shapes defined in **4.1.3.1** and **4.1.3.2** are applicable to overhanging crests also, for the ratio $M/H_d > 0.6$.

4.2 Discharge Computations

4.2.1 *Coefficient of Discharge*

The discharge over the spillway may be computed from the basic equation:

$$Q = \frac{2}{3}\sqrt{2g}.\ C.L'H^{3/2}$$

4.2.2 The non-dimensional coefficient of discharge has a theoretical minimum value of $\pi / (\pi + 2) = 0.611$ and a practical upper limit of about 0.75. The parameter $\frac{2}{3}\sqrt{2g}\ C$ is often called C_d which, however, is a dimensional quantity. The value of C_d generally varies from 1.80 to 2.21 (SI units).

4.2.3 The value of the coefficient of discharge depends on the following:

a) shape of the crest;
b) depth of overflow in relation to design head;
c) depth of approach;
d) extent of submergence due to tail water;
e) inclination of the upstream face; and
f) approach flow condition.

4.2.4 Figure 3 gives the coefficient of discharge C for the design head as a function of approach depth and inclination of upstream face of the spillway. These curves may be used for preliminary design purpose.

4.2.5 Figure 4 gives the variation of coefficient of discharge as a function of ratio of the actual head to the design head (H / H_d). This curve may be used to estimate C_d for heads other than design head H_d.

4.2.6 The coefficient of discharge is reduced due to submergence by the tail water. The position of the downstream apron relative to the crest level also has an effect on the discharge coefficient. Figures 5A and 5B give the variation of C_d with the above parameters.

4.3 Effective Length of Overflow Crest

4.3.1 The net length of overflow crest is reduced due to contractions caused by the abutments and crest piers. The effective length L of the crest may be calculated as follows:

$$L = L' - 2\,H\,(N.K_p + K_a)$$

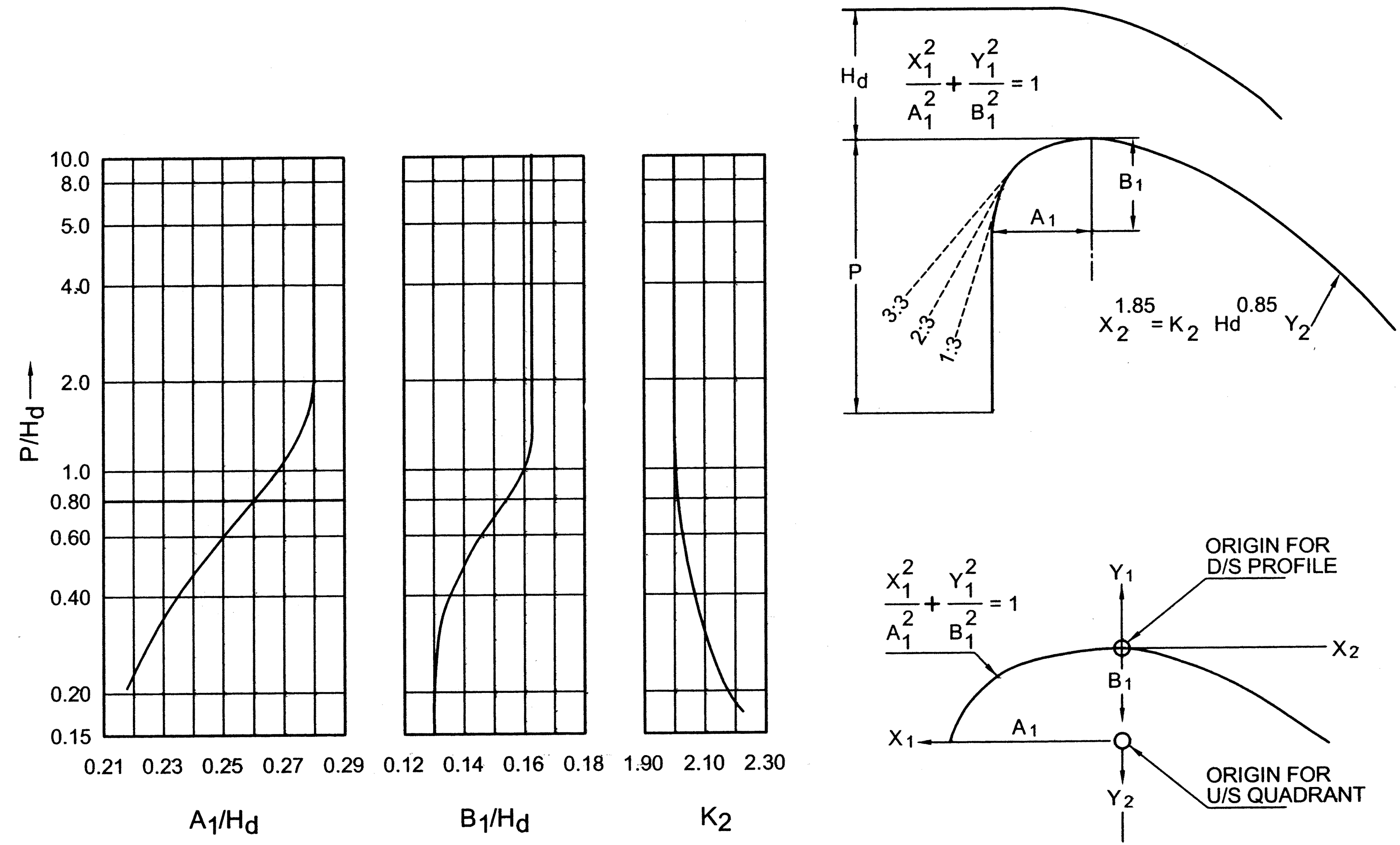


FIG. 1 OVERFLOW SPILLWAY CREST — DESIGN PARAMETERS

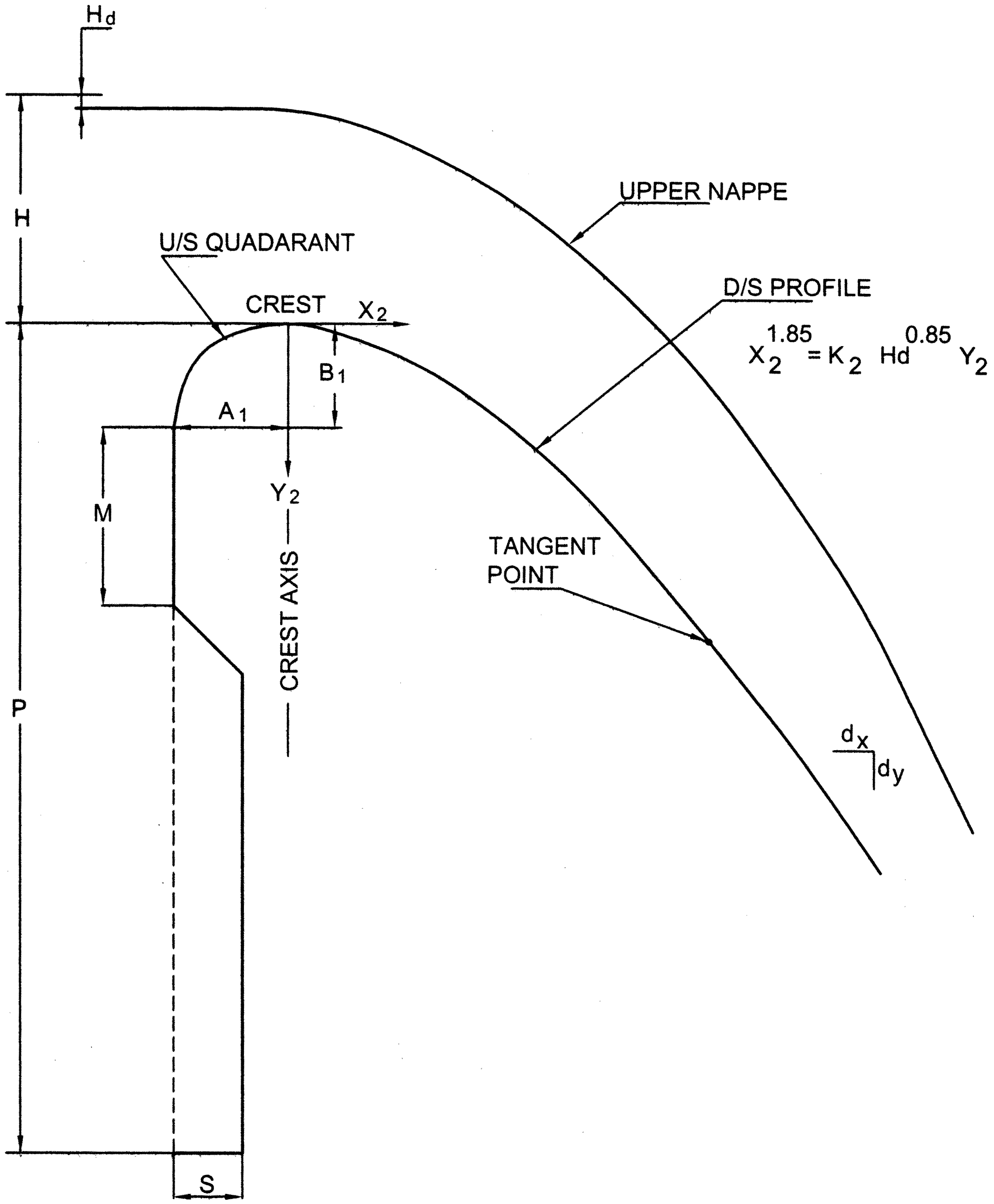


FIG. 2 OVERFLOW SPILLWAY

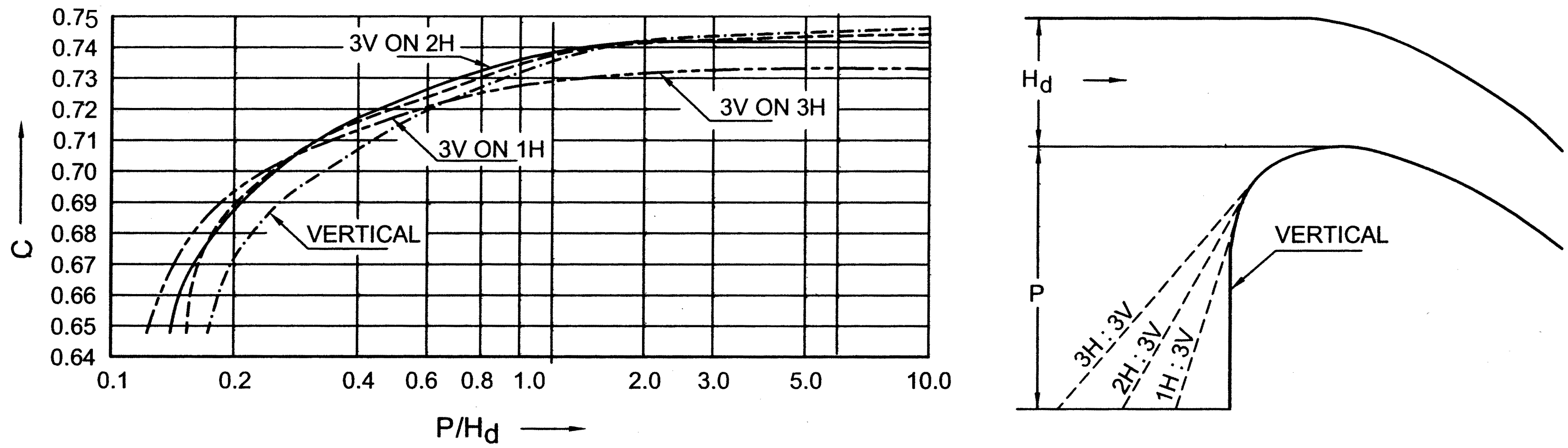


FIG. 3 DISCHARGE COEFFICIENT FOR DESIGN HEAD

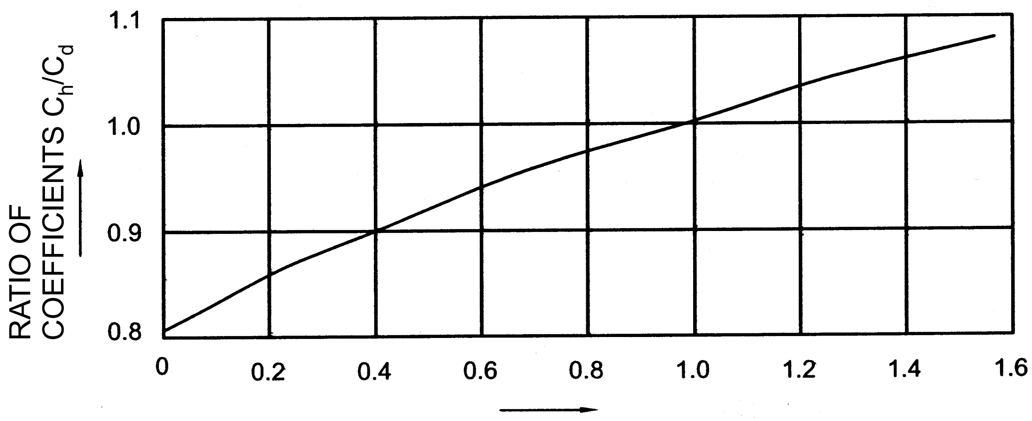
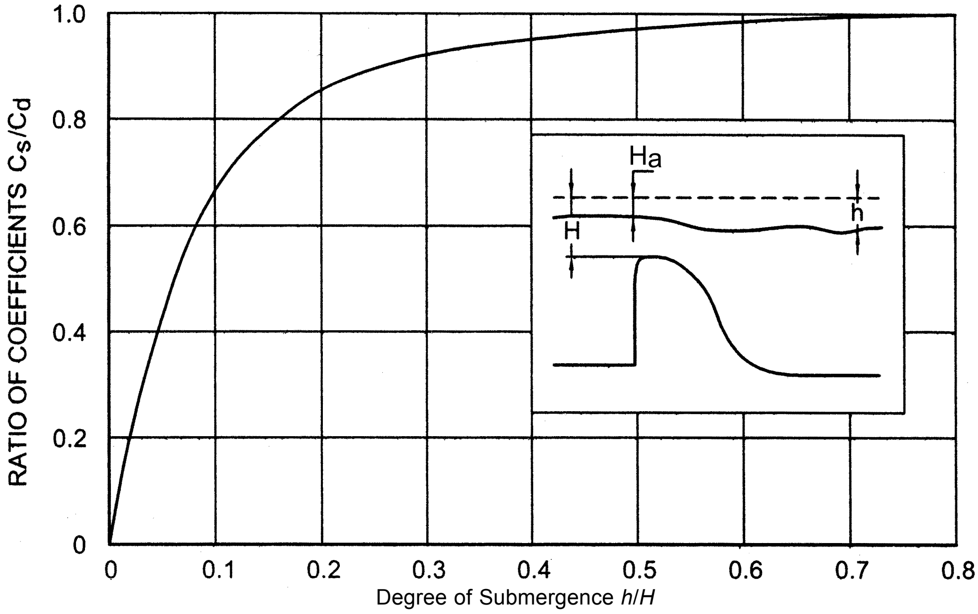
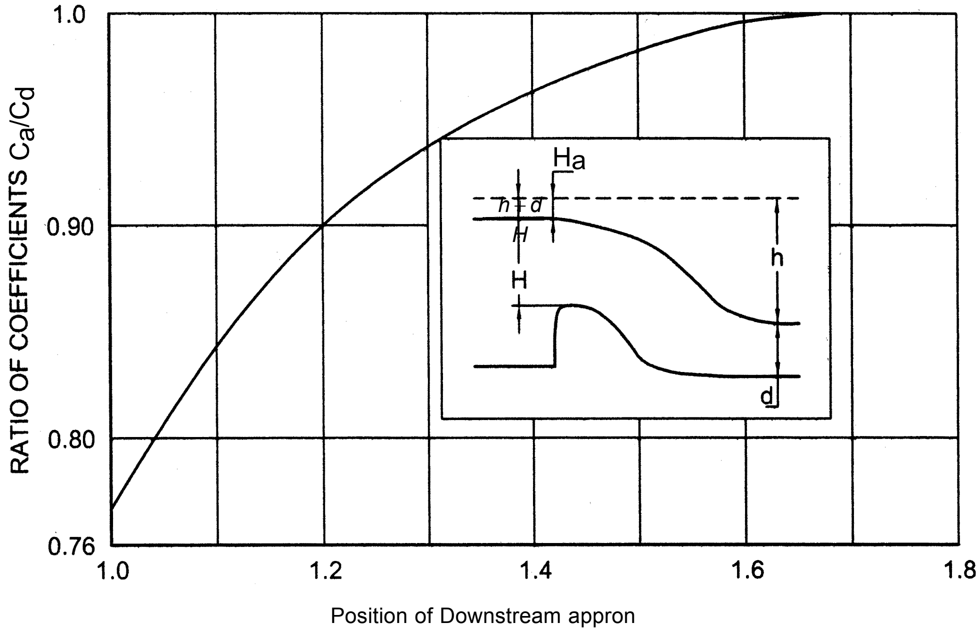


FIG. 4 RATIO OF HEAD ON CREST TO DESIGN HEAD (H/H_d)



5A Effect of Tail Water on Discharge Coefficients



5B Effect of Appron Elevation on Discharge Coeeficients

FIG. 5 COEFFICIENT OF DISCHARGE

4.3.2 The pier contraction coefficient, K_p is affected by the shape and location of the pier nose, thickness of the pier, the head in relation to the design head and the approach velocity. Average pier contraction coefficients may be taken as follows:

Type	K_p
For square-nosed piers with rounded corners on a radius of about 0.1 times the pier thickness	0.02
For round-nosed piers	0.01
For pointed-nosed piers	0

4.3.3 The abutment contraction coefficient is affected by the shape of the abutment, the angle between the upstream approach wall and the axis of flow, the head in relation to design head and the approach velocity.

Average abutment contraction coefficient may be taken as follows:

Type	K_a
For square abutments with head wall at 90° to direction of flow	0.20
For rounded abutments with head wall at 90° to direction of flow, when $0.5\ H_d > R > 0.15\ H_d$	0.10
For rounded abutments where $R > 0.5\ H_d$ and head wall is placed not more than 45° to the direction of flow	0

4.4 Determination of Design Head

Designing the crest profile for a particular head H_d results in a profile conforming to the lower nappe of a fully ventilated sharp crested weir and hence the pressures on the profile for the head H, are atmospheric. Operating the spillway for heads lower than H_d would give pressures higher than atmospheric and for heads higher than H_d the pressure would be sub-atmospheric. At the same time the coefficient of discharge would be reduced or increased (relative to that for the design head) for the heads lower or higher than the design head. Generally, designing the profile for a head lower than the highest anticipated head results in a steeper profile provided the sub atmospheric pressures could be kept within acceptable limits so as not to induce cavitation. The ratio of actual head to design head (H/H_d) for ensuring cavitation-free performance of the spillway crest is a function of design head H_d. The extent of sub-atmospheric pressure for an under designed spillway profile shall be ascertained from hydraulic model studies for the specific case. Generally design head is kept as 80 to 90 percent of the maximum head.

5 OGEE PROFILE FOR GATED SPILLWAY

5.1 Shape of the Profile

5.1.1 When spillways are equipped with gates (the most common type of gate is radial gate), discharges for partial gate openings will occur as orifice flow. With full head on the gate and with the gate partially opened the jet emerging from the gate will be in the form of a trajectory conforming to a parabola

$$X^2 = 4HY$$

If sub-atmospheric pressures are to be avoided along the crest, the shape of ogee downstream from the gate sill should conform to the trajectory profile. The adoption of a trajectory profile rather than a nappe profile will result in a flatter profile and reduced discharge efficiency under full gate opening. Where the discharge efficiency is not important and a flatter profile is needed from consideration of structural stability, the trajectory profile may be adopted to avoid sub-atmospheric pressures along the crest. When the ogee is shaped to the ideal nappe profile for the maximum head (*see* **4.1.3.1** and **4.1.3.2**), sub-atmospheric pressures would occur in the region immediately downstream of the gate for small gate openings. The magnitude and area of sub-atmospheric pressures may be minimized by placing the gate sill 0.3 m to 1.0 m below the crest level, downstream of the crest axis. Experiments have shown that under such a condition the minimum crest pressures may range from about 0.1 H_d for upstream water level at design head to about 0.2 H_d for heads about 1.3 H_d. The ogee profile may thus be designed considering the magnitude of the minimum pressures.

5.2 Discharge Computation

5.2.1 The discharge for a gates ogee crest at partial gate opening is similar to flow through a low-head orifice and may be computed by the equation:

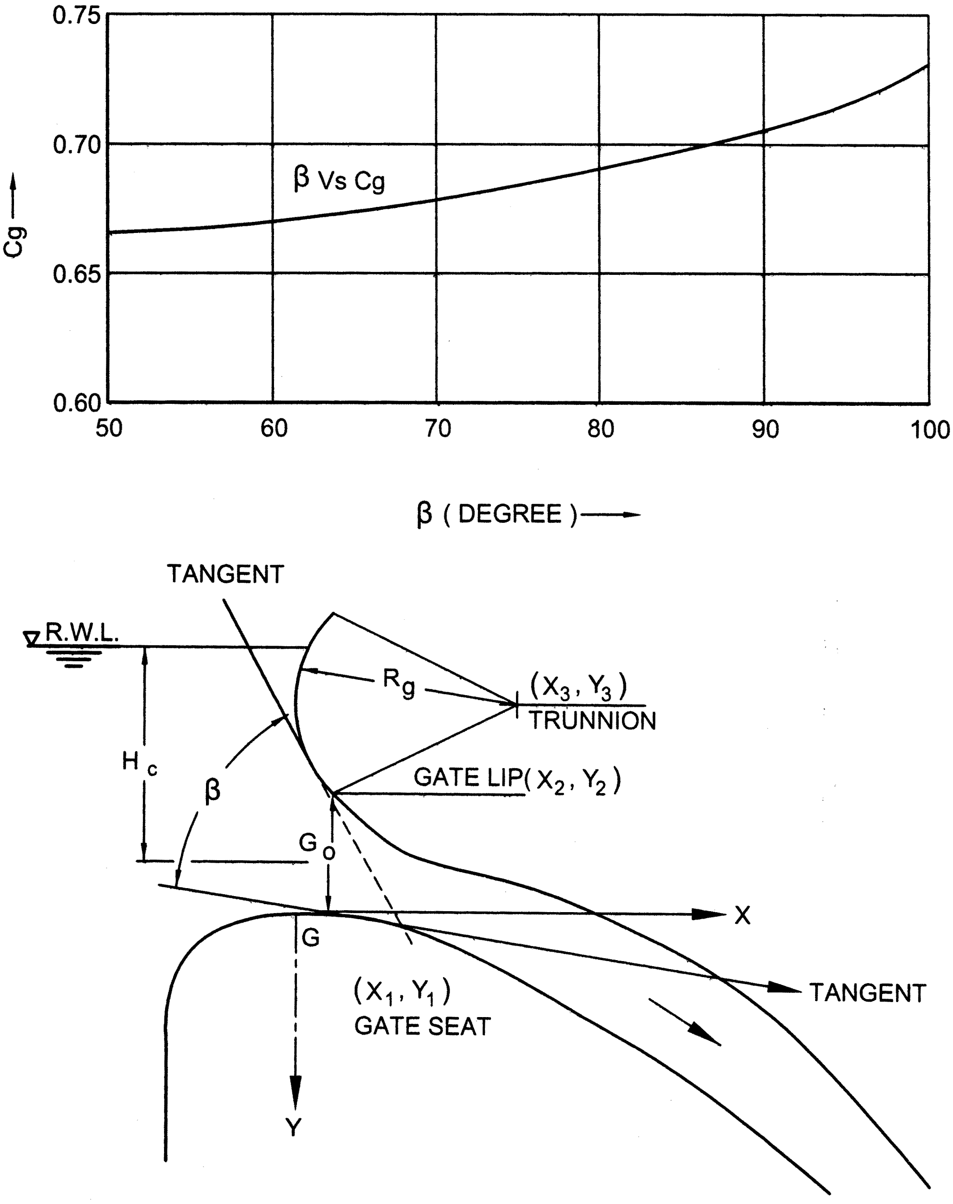
$$Q = C_g\, G_o\, L\sqrt{2gH_c}$$

The coefficient C_g differs with different gate and crest arrangements and is influenced by the approach and downstream conditions. Figure 6 shows coefficient of discharge for flow under the gate for various ratios of gate opening to total head. The curve presents average determined for various approach and downstream conditions and may be used for preliminary design purpose.

6 HYDRAULIC DESIGN ASPECTS OF ORIFICE SPILLWAY

6.1 General

Orifice spillways combine the advantage of greater



$$Q = C_g\, G_o\, L\sqrt{2gH_c}$$

FIG. 6 COEFFICIENT OF DISCHARGE FOR FLOW UNDER GATE

depth of flow over the crest and moderately sized gates – an arrangement made possible by inclusion of a breast wall. The spillway would allow the setting of its crest at significantly lower elevation, yet retaining the choice of a high dam for creating head for power generation. Orifice spillways have been widely recognized as the most appropriate especially for run-of-the-river projects for handling both flood releases and flushing of sediment. Orifice spillway is essentially large capacity outlet provided in the dam and controlled by gates as shown in Fig. 7.

The following parameters are required to be determined:

a) Bottom profile of the spillway crest including the upstream and down stream quadrants,
b) Roof profile of the orifice opening,
c) Estimation of discharge characteristics of the spillway,
d) Size and dimensions of the orifice spillway,
e) Protection of the spillway surface to resist abrasion, and
f) Special considerations for the energy dissipator.

Table 1 gives the salient features of the orifice spillway for existing structures.

6.2 Design of Spillway Crest Profile

The hydraulic behavior of orifice spillway changes with the varying reservoir levels. The flow is free flow for reservoir water levels below the top of the orifice opening. For higher water levels the flow is orifice flow. The spillway crest profile is required to be designed for orifice (pressurized) flow.

6.2.1 *Upstream Quadrant*

The upstream quadrant may conform to an ellipse similar to ogee profile of the free overflow spillway mentioned at **4.1.3.1**.

6.2.2 *Downstream Profile*

The downstream profile is flatter as compared to the overflow crest profile to avoid flow separation and negative pressures on the crest for small partial gate openings. The crest profile generally follows the equation $x^2 = 4\ H_c\ y$, where H_c is the head over the centerline of the orifice opening.

6.3 Design of Roof Profile of Orifice

Hydraulic design of roof profile of orifice opening is very important because it guides the flow smoothly. This governs the coefficient of discharge of the spillway. This profile should be simple to construct and the pressures on the profile should not be excessively negative. Usually, a profile in the form of part/full quarter of an ellipse is provided bearing the equation

$$\frac{x^2}{A_2^2} + \frac{y^2}{B_2^2} = 1$$

where

A_2 = width of semi-major axis,

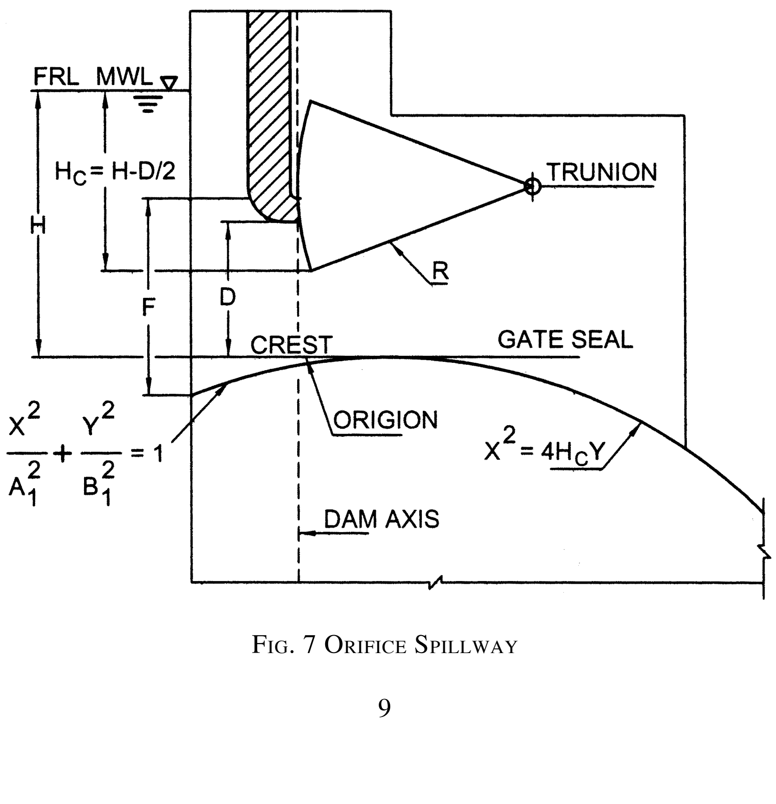


Fig. 7 Orifice Spillway

B_2 = width of semi-minor axis which governs the steepness of the profile, and

x and y are the coordinates of the profile.

Figure 8 shows a typical roof profile of the orifice opening. Usually steep profiles yield increased coefficient of discharge, whereas flat profiles tend to reduce the discharging capacity. However, negative pressures increase as the profile becomes steeper. The roof profiles of orifice opening are usually steel lined/ constructed in high strength concrete to avoid cavitation damage.

6.4 Discharge Characteristics of Orifice Spillway

The spillway operates as free overflow spillway for lower discharges, whereas for higher discharges the flow through the spillway is governed by the orifice flow. Generally, the orifice flow condition requires head over the crest in excess of about 1.5 to 1.7 D, where D is the height of the orifice opening.

For free flow conditions the discharge is given by

$$Q = \frac{2}{3}\sqrt{2g}\,C\,L\,H^{3/2}$$

where

Q = discharge, in cumec;

C = non-dimensional discharge coefficient;

L = total effective length of spillway, in m; and

H = head over the crest including velocity head, in m.

For orifice flow

$$Q = C_d . n . A . \sqrt{2g(H_c)}$$

where

Q = discharge, in cumec;

n = number of orifices;

A = area of orifice, in m^2;

$H_c = H - \frac{D}{2}$ = head over the center line of orifice; and

C_d = Discharge Coefficient for Design head.

Figure 7 shows the definition sketch for calculation of discharging capacity. The coefficient of discharge for orifice flow is influenced by the entrance profile – composed by the roof profile of the orifice, spillway crest profile and side wall profiles if provided. The coefficient of discharge for the orifice flow is generally in the range of 0.7 to 0.85 (Table 1). Figure 9 shows a typical discharging capacity curve with full and partial gate openings.

6.5 Size of the Orifice Spillway

Flushing used to be carried out previously by providing small sluices of the size of 3 m × 4 m or so at very low level. However, it was realized that these sluices were effective only locally. Also, there was a tendency of choking of sluices within a short period. Large openings of the size of 6 - 15 m (W) × 10 - 21 m (H) are required to be located 30 - 40 m below the full reservoir water level and as near the river bed as possible for flushing of the reservoir.

6.6 Protection of Spillway Surface

The large velocities associated with the high heads may increase the potential for cavitation and erosion damage to the structure. Adequate protection measures should be taken during the construction, to withstand the erosive power of the silt laden water while flushing

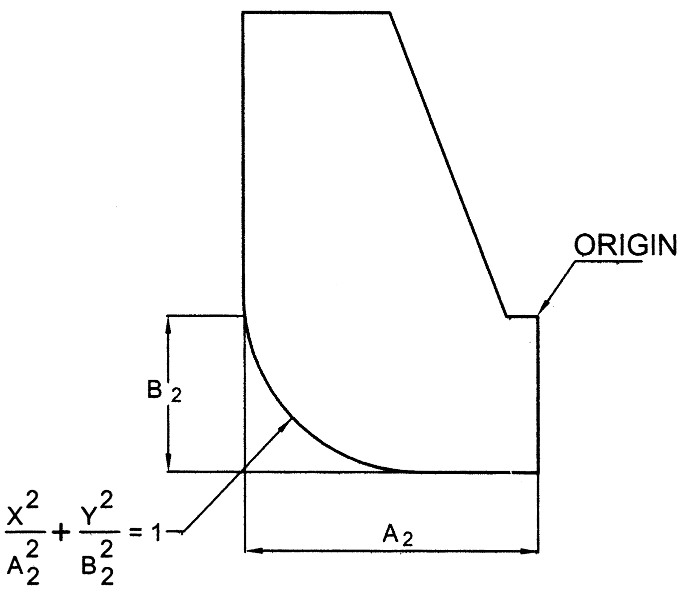


FIG. 8 ROOF PROFILE OF THE ORIFICE OPENING

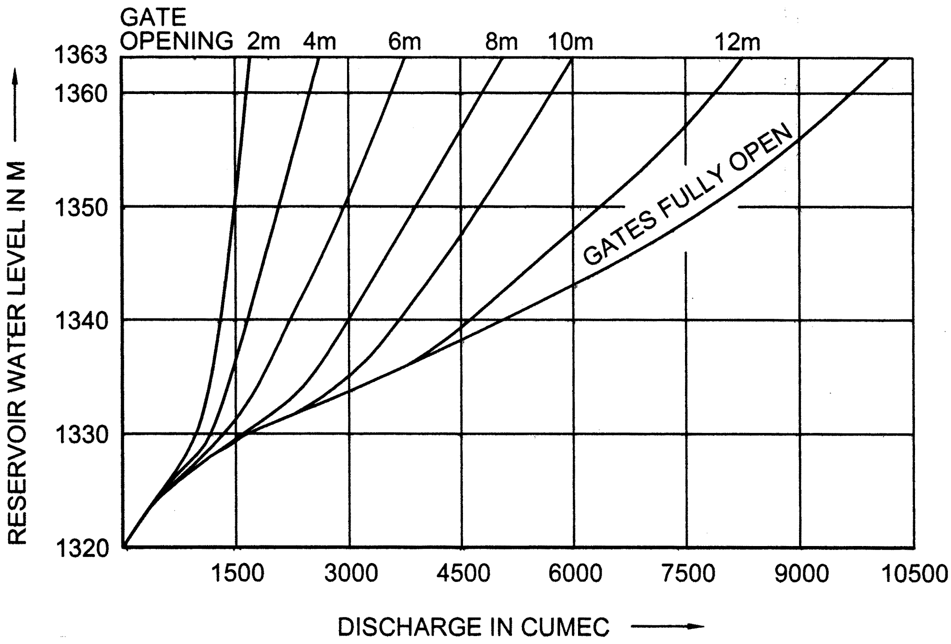


FIG. 9 A TYPICAL DISCHARGING CAPACITY CURVE FOR AN ORIFICE SPILLWAY

the reservoir and flood routing, by way of special type of concreting.

6.7 Special Considerations for Energy Dissipator

Special considerations are required for design of suitable energy dissipator, since the spillway has to surpass both the flood and the sediment. Ski-jump bucket is found to be the most suitable as energy dissipator because of its obvious advantage during flushing operation. The sediment passes down the spillway with supercritical flow without deposition and churning in the bucket. Fortunately, steep bed slopes of the rivers in the hilly regions result in low tail water depth permitting this type of energy dissipator. A hydraulic jump stilling basin may have to be adopted where geological conditions are not favorable or tail water levels are high. Because of the requirement of passing high sediment flows, use of energy dissipating appurtenances like chute and baffle blocks is not advisable. As a result the stilling basin becomes excessively long and often deep-seated below the general river bed, making it vulnerable to deposition of silt during flushing operation. A trade-off is desirable between the hydraulic efficiency of energy dissipation and the self-cleansing potential of the stilling basin. Cylindrical end sills are generally preferred for easy movement of sediment out of the basin. A concrete apron downstream of the end sill is required to protect the spillway against undermining. Provision of roller bucket is generally avoided as an energy dissipator due to likelihood of abrasion damage of the bucket due to churning of sediment.

7 MODEL STUDIES

The guidelines given above would be useful in preparing a preliminary design of high ogee overflow and orifice spillway. The final design should be evolved on the basis of studies on a hydraulic model for discharging capacity, pressures on spillway surface and breast wall etc.

Table 1 Details of Bottom Profile of Breast Wall for Spillway
(*Clauses* 6.1 *and* 6.4)

Sl No.	Name of Project	Spillway Profiles		Breast Wall	F	D	H	P	Span		Discharge	Discharge Intensity	C_d
		Upstream	Downstream	Bottom Profile	m	m	m	m	Nos.	Width (m)	Cumec	Cumec/m	
i)	Chamera - I	Combination of circular arcs of R = 5.2 m and 13 m	$x^2 = 102\ y$	$\frac{x^2}{6^2}+\frac{y^2}{2^2}=1$	17.77	12.5	32.5	110	8	10	22 000	275	0.83
ii)	Chamera III	Circular arc of radius 26.27 m	$x^2 = 122\ y$	$\frac{x^2}{7^2}+\frac{y^2}{3.5^2}=1$	37.96	16.5	37	20	3	12.5	11 400	304	0.78
iii)	Kurichu	$\frac{x^2}{4.5^2}+\frac{y^2}{2.5^2}=1$	$x^2 = 80\ y$	$\frac{x^2}{5.5^2}+\frac{y^2}{2^2}=1$	24.25	14	28	26	5	10.5	12 200	232	0.83
iv)	Nathpa Jhakri	Flat	$x^2 = 126\ y$	$\frac{x^2}{8.5^2}+\frac{y^2}{2.833^2}=1$	11.33	8.5	37.5	23	5	8.5	7 200	169.41	0.78
v)	Nimoobazgo	$\frac{x^2}{5^2}+\frac{y^2}{2^2}=1$	$x^2 = 100\ y$	$\frac{x^2}{5.6^2}+\frac{y^2}{2^2}=1$	11	9	23.5	28	5	7	4 500	128	0.84
vi)	Pandoh	Flat	$x^2 = 4\ 273.5\ y$	$\frac{x^2}{39.37^2}+\frac{y^2}{13.1^2}=1$	16	13	21.64		5	12	9 939	166	0.73
vii)	Parbati II	$\frac{x^2}{7.38^2}+\frac{y^2}{3.5^2}=1$	$x^2 = 101\ y$	$\frac{x^2}{5^2}+\frac{y^2}{2^2}=1$	14.5	9	33	36	3	6	1 850	102.77	0.81
viii)	Parbati III	$\frac{x^2}{9.86^2}+\frac{y^2}{5.344^2}=1$	$x^2 = 100\ y$	$x = 0.158y^{2.4}$	17.64	14	32	10	2	7.2	3 300	157.14	0.74
ix)	Sewa II	$\frac{x^2}{8.059^2}+\frac{y^2}{4.152^2}=1$	$x^2 = 96.4\ y$	$\frac{x^2}{3.6^2}+\frac{y^2}{2^2}=1$	14.95	10.8	29.5	9.7	4	7	4 020	143.57	0.80
x)	Subansiri	$\frac{x^2}{5^2}+\frac{y^2}{2^2}=1$	$x^2 = 195\ y$	Width = 5m	19.2	14.7	63.25	51	9	11.5	35 000	338.16	0.72
xi)	Teesta IV	$\frac{x^2}{6^2}+\frac{y^2}{3.5^2}=1$	$x^2 = 67\ y$	$\frac{x^2}{4.25^2}+\frac{y^2}{2^2}=1$	22.1	17	25.25	6	7	11	15 400	200	0.72
xii)	Teesta V	$\frac{x^2}{6^2}+\frac{y^2}{3.5^2}=1$	$x^{1.85} = 45\ y$	$\frac{x^2}{6^2}+\frac{y^2}{2^2}=1$	17.5	12	40.72	25	5	9	9 500	211.11	0.76
xiii)	Uri II	$\frac{x^2}{5.4^2}+\frac{y^2}{12.5^2}=1$	$x^2 = 80\ y$	$\frac{x^2}{4.8^2}+\frac{y^2}{2^2}=1$	14.65	11.4	24	20	4	9	4 850	134.72	0.81

NOTE — The data given in Table 1 is a compiled data of model studies for 13 projects. It may be used as a guideline for the preliminary design and design may be finalized from hydraulic model studies.

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Amendments Issued Since Publication

Amend No.	Date of Issue	Text Affected

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